STIFFNESS ANALYSIS OF
THE ACADEMY TOWERS BUILDING
HONOLULU, HAWAII

UNIVERSITY OF HAWAII
DEPARTMENT OF CIVIL ENGINEERING

DECEMBER 1974

HONORS 493-494

by

KIN LEK CHAN

Thesis Adviser
Dr. George T. Taoka

ABSTRACT

The Academy Towers Building is located at the corner of Ward Avenue and Green Street in Honolulu, Hawaii.It is a twenty-seven (27) story building consisting of two dwelling units per floor.

A special structural feature of this building is that there is a very stiff beam on the roof of the building. This increases the stiffness of the building thus reducing the lateral displacement of different floors due to lateral forces.

Mr. Peter K.W. Lum, a former honorstudent at the university of Hawaii did a study on 'Lateral Load Analysis' for the building. His paper included the lateral displacement of different floors due to wind load and earthquake load. He also computed the displacement of different floors due to wind load assuming that the stiff beam on the roof was not built. It is concluded in his report that the stiff beam on the roof reduces the lateral displacement due to wind load by approximately 30%.

A stiffness analysis of the structure is the basis of my investigation. The displacement of all floors are computed as the building is subjected to wind or

earthquake. Wind load and earthquake load are computed according to the 1973 uniform Building Code, Part VI, Chapter 23, on general design requirements. The period of the building is also computed. The same analysis is done again on the same building with the assumption that there is no stiff beam on the roof.

It is concluded that the stiff beam on the roof reduces the displacement due to earthquake load by 40% and also the period is also reduced by 10% to 20%.

LIST OF TABLES

Table		Page
1	Areas and moments of inertia of columns and walls	A21
2	Deflections due to wind load in east-west direction (with stiff beam)	A22
3	Deflections due to wind load in east-west direction (without stiff beam)	A23
4	Deflections due to wind load in north-south direction (with stiff beam)	A24
5	Deflections due to wind load in north-south direction (without stiff beam)	A25
6	Deflections due to earthquake load in east-west direction (with stiff beam)	A26
7	Deflections due to earthquake load in east-west direction (without stiff beam)	A27
8	Deflections due to earthquake load in N-S direction (with stiff beam)	A28
9	Deflections due to earthquake load in N-S direction (without stiff beam)	A29
10	Fundamental mode shape in east-west direction (with stiff beam) by Rayleigh 's method	A30

Table		Page
11	Fundamental mode shape in east- west direction (without stiff beam) by Rayleigh 's method	A31
12	Fundamental mode shape in north-south direction (with stiff beam) by STRUDL	A32
13	Fundamental mode shape in north-south direction (without stiff beam) by STRUDL	A33

LIST OF FIGURES

Figure		Page
1	Columns and walls dimensions	A3
2	Location of centroid	A4
3	Subdivision into three frames	A5
4	Subdivision into two frames	A6
5	Analysis of stiff beam as equivalent T-beam for east-west direction	A7
6	Analysis of stiff beam as equivalent double T-beam for N-S direction	A8
7	Wind velocity profiles	A9
8	Wind load distribution	A10
9	Wind analysis	A11
10	Earthquake analysis	A12
11	Earthquake load distribution	A13
12	Map for seismic probability zones	A14
13	Deflections due to wind load (E-W)	A15
14	Deflections due to wind load (N-S)	A16
15	Deflections due to earthquake (E-W)	A17
16	Deflections due to earthquake (N-S)	A18
17	Joints and members for N-S direction	A19
18	Joints and members for E-W direction	A20

TABLE OF CONTENTS

		Page
ABSTRACT		i
LIST OF TABLES		iii
LIST OF FIGURES		v
CHAPTER 1	INTRODUCTION	1
	1-1 Objectives and scope of study	1
	1-2 Features of the building	2
CHAPTER 2	COLUMNS AND SHEAR WALLS	4
	2-1 Columns	4
	2-2 Shear walls	4
	2-3 Moments of inertia and stiffness	6
	2-4 Simplified approach inanalysis	6
CHAPTER 3	DEAD LOADS	10
	3-1 Dead load calculation	10
CHAPTER 4	WIND ANALYSIS	12
	4-1 Wind load	12
	4-2 Variation of wind velocity with elevation	13
	4-3 Design requirement for wind	14
	4-4 Result of analysis	15

Page

CHAPTER 5 EARTHQUAKE ANALYSIS 16

5-1 Design against earthquake 16

5-2 Design requirement for earthquake... 17

5-3 Earthquake load 18

5-4 Result of analysis 18

CHAPTER 6 DYNAMIC ANALYSIS 19

6-1 Rayleigh 's Method 19

6-2 STRUDL-II 20

CHAPTER 7 EFFECT OF STIFF BEAMS ON ROOF 21

7-1 Comparsion of analyses 21

CHAPTER 8 CONCLUSION, DISCUSSION AND RECOMMANDATION 23

8-1 Discussion and recommandation for wind analysis 23

8-2 Discussion and recommandation for earthquake analysis 25

8-3 Conclusion 27

REFERENCES 28

APPENDIX A A1

FIGURES A3

TABLES A21

SP

CHAPTER 1

INTRODUCTION

Due to the current trend toward taller structures, designers are more aware of the dynamic effects of lateral loadings such as wind loads and earthquake loads. The designed structures should be able to withstand vertical loads as well as to be able to resist lateral forces. All the requirements as specified in the Uniform Building Code should be met in the design.

The Academy Towers Building is analyzed in this report. It was designed by Mr. Richard Libbey, a consulting engineer in Honolulu, Hawaii. The author wishes to express his thanks to Mr. Libbey for providing all the information which made this study possible.

1-1 Objective and Scope of Study

Wind, earthquake and dynamic analyses are the subject of this study. Wind loads and earthquake loads are computed according to the design requirements as specified in the 1973 Uniform Building Code.

The object of the study is to investigate the lateral displacement of all floors when the building is subjected to wind and earthquake loads. The natural period of the building is also computed. The analysis is then compared with similar analysis assuming the stiff beam at the top is absent.

1-2 Features of the building

The building consists of rectangular columns and shear walls which function as a unit under lateral loading. The plan view shows the arrangements and dimensions of columns and shear walls. (see Fig. 1) The core of the building is the elevator shaft which is located near the centroid of the building. (see Fig. 2) This arrangement will minimize torsional effects.

A very interesting feature of the building is that there is an extremely beam located on the roof which affects horizontal displacement under lateral loading. (see Chapter 7 for more discussion)

The concrete strength varies with the height of the building. For tall buildings, the lower floors are required to withstand a much larger vertical load and also resist a larger bending moment as compared to those of the upper floors. That is why engineers use concrete of different strength to meet the requirements.

For this building, the concrete strength varies as shown in the table on the next page.

Floor	f'c ($\overset{K}{\cancel{p}}$si)	E ($\overset{K}{\cancel{p}}$si)
Bsmt to 4th floor	5,000	4,286.83
5th to 8th floor	4,000	3834.25
9th to roof	3,750	3,712.00

Bsmt --- basement

f'c --- strength of concrete

E --- modulus of Elasticity

Modulus of elasticity of concrete is defined in ACI Code, 8.3.1 as the following.

$$E = W^{1.5} \times 33\sqrt{f'c}$$

W --- unit weight of concrete, 150 pounds per cu. ft. is used.

CHAPTER 2

COLUMNS AND SHEAR WALLS

2-1 Columns

Columns are members which usually carry axial compression loads. Inthe design of tall buildings, the columns must be able to withstand vertical load. Wind load may induce critical stresses and failure occurs due to buckling and compression. Designers must take wind load into account in the design of a tall building.

2-2 Shear walls

Shear wall is a structural system providing stability against wind, earth tremors or blasts. Such a system may be constructed in steel or concrete and may either be solid or perforated. The proforation should be arranged in a way that the stiffness depends on the whole system instead of its individual elements. The system can consists of a plan wall, part of a curved wall, a closed loop, a rectangular box of a system of concentric or eccentric cores. In this case, it is a plane wall.

Shear walls serve three purposes:

(I) To withstand vertical loads

As far as vertical load is concerned, shear wall is just like a column transmitting vertical loads to the foundation.

(II) To resist lateral load

This is the main reason of using shear walls. The floor slabs act as diaphragms distributing the horizontal loads to the vertical stiff shear walls which in turn transmit the loads to the foundation. The foundation is required to distribute highly concentrated loads over a sufficient area to prevent overstressing the soil.

The advantage of using shear wall is to resist lateral load. As compared to a column, shear wall provides more stiffness because of a larger moment of interia. A larger moment of inertia gives a smaller deflected curvature, this leads to a smaller deflection. That is why shaer wall is better than a column in resisting lateral loads.

(III) To make a more or less permanent division for rooms

In order to divide the space into dwelling units, shear walls are arranged in such a way to serve more or less permanent divisions for rooms for residents in this condominium. In this case, two dwelling units per floor is designed.

2-3 Areas, moments of inertia and stiffness

In order to compute the deflections of different floors of the building, the areas and moments of inertia of columns and shear walls are required.

As in table 1, mainly wall #7, #5, #6 and #10 resist wind from north or south since their moments of inertia are larger than those of the rest. And mainly wall#4 and #6 resist wind from east or west since their moments of inertia are larger among the rest. Larger moments of interia corresponds to larger stiffness. The reason of combining columns and shear walls of different stiffness is to enable the entire structure to function as a whole unit to resist lateral loads.

2-4 Simplified approach in Analysis

For simplicity, the structure is analysed by means of two plane frames as shown in Fig. 3and Fig. 4.

In the east-west direction, the frame consists of three vertical members , the neutral axes are located at the centroids of the two wall #4 and wall #6. By using the formula of area moment transformation, the moments of inertia of other columns and walls are transformed to the three neutral axes.

Area-moment transformation :

$$I_c = I + Ad^2$$

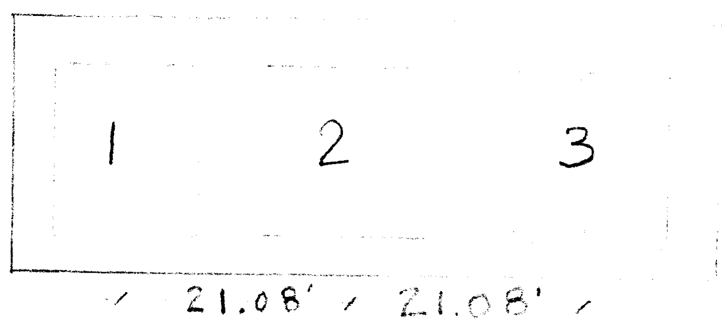
I_c -- transformed moment of inertia

I -- moment of inertia about its own axis

A -- area of column or wall

d -- distance between the neutral axes

In summary, for east-west direction :



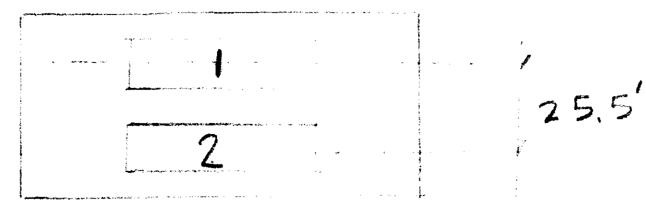
Floor	Part	Area (ft^2)	IZ (ft^4)
Bsmt to 4th	1	46.11	4,195.38
	2	54.20	1,572.12
	3	46.11	4,195.38
5th to roof	1	46.11	4,195.38
	2	51.20	1,464.38
	3	46.11	4,195.38

IZ -- total moment of inertia

Bsmt -- basement

In the north-south direction, the frame consists of two vertical members; the neutral axes are located at the centroids of column #1 and wall #7. The moments of inertia of other columns and walls are transformed to the two axes (neutral axes) by area moment transformation.

In summary, for north-south direction :



Floor	part	Area (ft^2)	IZ (ft^4)
Bsmt to 4th	1	71.00	3,670.00
	2	94.61	4,676.70
5th to roof	1	68.00	3,650.00
	2	94.61	4,676.70

The frame in east-west direction consists of three composite shear walls as vertical members while the frame in the north-south direction consists of two composite shear walls as vertical members. Instead of analysing the three dimensional structure, two modified, plane frames are analysed. Slabs are horizontal members. The thickness is five inches. The areas and moments of inertia are computed as shown in the following table.

Direction	Area (ft^2)	IZ(ft^4)
East-West	18.33	0.265
North-South	32.08	0.464

Loads (lateral) are computed as concentrated loads and assumed to be applied at each floor slab.

A computer programn, STRUDL-II, is used in the analysis. By means of a computer (IBM 360 was used), the deflections of the modified frames due to lateral loads are computed. (See Appendix A for the application of the programn STRUDL--II)

CHAPTER 3

DEAD LOAD

1973 Uniform Building Code defines dead load as the vertical load due to the weight of all permanent structural and non-structural components of a building, such as walls, floors, roofs and fixed service equipments.

For simplicity, the dead load is the weight of the main structural components such as columns, shear walls and slabs, the weight of reinforcing steel bars and other non-structural components of the building are neglected.

3-1 Dead Load Calculation

The weight of the building depends on the unit weight (or density) of concrete. The unit weight of concrete depends on how the concrete is made. For simplicity, 150 pounds per cubic foot is assumed to be the unit weight of concrete used.

For basement to 4th floor :

	Area (ft^2)	Volume(ft^3)	weight(kips)
Columns	146.38	1,171.04	175.66
slabs	3388.0	1,411.66	211.75
Railings	35.33	141.33	21.20

Total weight per floor is 408.61 kips

For 5th floor to RGU :

	Area(ft^2)	Volume(ft^3)	Weight(kips)
Columns	143.88	1,147.04	172.06
slabs	3388.0	1,411.66	211.75
Railings	35.33	141.33	21.20

Total weight per floor is 405.01 kips.

For Machine Room to Roof :

	Weight
Machine room ------------	117.75 kips
Stiff beam in east-west direction --------	84.00 kips
Stiff beam in north-south direction --------	45.32 kips

Total weight from machine room to roof is 247.07 kips.

The total dead load of the entire building is 11,200 kips.

CHAPTER 4

WIND ANALYSIS

Wind is air in motion, induced and maintained by temperature differences arising from uneqal heating of the earth 's surface by the sun. The velocity and direction of wind are affected primarily by the rotation of the earth on its axis. It would take a lengthy discourse in meterology to thoroughly explain the complicated nature of wind . Structural engineers are interested in the velocity of wind on the surface of the earth. It is because wind load mainly depends on the velocity of wind.

4-1 WIND LOAD

The evaluation of the effect of wind on an object in its path is a complex problem. For simplicity, Bernoulli 's equation for stream line flow can be used to determine the local pressure at the point as a column of air strikes (at 90°) a stationary body. The assumption is that air is non-compressible and non-vicous, this is reasonable for the magnitude of velocity of air for which most structures are designed.

Bernoulli 's equation : $Q = \frac{1}{2} d v^2$

Q -- wind pressure, d -- density of air,

v -- velocity of air

4-2 VARIATION OF WIND VELOCITY WITH ELEVATION

Since wind load (pressure) depends on the square of the velocity of wind, the variation of wind velocity with height must be evaluated. The flow of air close to the ground surface is slowed by surface roughness which depends on the size, height and density of the buildings, vegetation etc. Fig. 7 shows velocity profile and corresponding exponent variation with height as suggested by Davenport.

The equation which is generally accepted is known as the power law. In fact, it has been adopted by U.S. Weather Bureau.

$$\frac{V}{v} = \frac{H}{h}^{p}$$

V -- velocity at height H

v -- velocity at height h

p -- a parameter

Various value for p has been suggested but 1/7 for open space is generally accepted. It should be noted that the power law gives only the average values of measured wind velocities.

4-3 DESIGN REQUIREMENT FOR WIND

Wind load is considered to be live load in reinforced concrete design. 1973 Uniform Building Code specified that the wind pressure shall be taken upon the gross area of the vertical projection of that portion of the building or structure measured above the average level of the adjoining ground.

From table No. 23F of 1973Uniform Building Code (Honolulu is No. 20 in Wind-Pressure Map), wind pressure for various height zones above ground are given as the following:

Less than		30 feet	----	15	psf
30	to	49 feet	----	20	psf
50	to	99 feet	----	25	psf
100	to	499 feet	----	30	psf

For simplicity, wind pressures are assumed and computed as concentrated forces acting at each floor slab (The computations are shown in Fig. 8 and Fig. 9). The loads used are modified loads.

4-4 RESULTS OF ANALYSIS

Apply the modified loadsto the modified frames and use a computer programn for computation, the computed deflections are tabulated as in Table 2 and Table 4.

The maxium horizontal displacements are 0.5697 inches and 1.128 inches for east-west direction and north-south direction respectively. For a twenty-seven story building (with a height of about ~~240~~ 230 feet), these displacements are insignificiant.

It is concluded that the Academy Tower has sufficient wind resistance to wind load.

CHAPTER 5

EARTHQUAKE ANALYSIS

Earthquake is the ground vibration induced by a sudden release of strain energy accumulated in the crust and upper mantle. It may happen in any part of the world, but earthquakes are move frequent and more violent in two great belts. One belt almost encircles the Pacific Ocean and the other stretches across Southern Asia into Mediterranean region. Hawaii falls into the first belt as mentioned above.

5-1 Design against Earthquake

There are a lot of factors affecting the design against earthquake. The following several factors are usually being considered.

1. Size and distance of earthquake:
    a. Vertical and horizontal distance between the structure and the epicenter.
    b. Displacement of earthquake.
2. Intensity:
    a. Amount of energy release.
    b. Time of lasting.
3. Site Resonance.
4. Soil dynamics.
5. Types of foundation.
6. Interaction between soil and structure.

7. Structure:

   a. Natural frequency of the structure.

   b. Ductility of the structure.

   c. Damping effect of the structure.

5-2 Design Requirement for Earthquake

The seismic design of buildings usually is carried out by one of the following methods.

1. By using equivalent static loadings to represent the actual dynamic actions.
2. By a dynamic analysis based an appropriate earthquake motion for the site and soil conditions.

The method of treating earthquake load as static lateral loading is used in Uniform Building Code. The minimum lateral load is calculated as the following:

$$V = Z\ K\ C\ W$$

$Z = 1.0,\ 0.5,\ 0.25$

$Z = 0.25$ for Ohau (See Fig. 12)

$K = 1.33,\ 1.0,\ 0.8,\ 0.67$

$K = 1.0$ is used. (from Uniform Building Code)

W = dead load of the building (See Chap. 3)

$C = \dfrac{0.05}{\sqrt[3]{T}}$ $\qquad T = \dfrac{0.05h}{\sqrt{D}}$

h = height of building

D = dimension parallel to seismic force in feet.

5-3 Earthquake Load

The loads are computed according to the Uniform Building Code requirement as described above. See Fig. 10 & 11. for computation.

5-4 Results of Analysis

Apply the load to each floor slab. And use a programn STRUDL-II, the deflections are computed.

The maximum horizontal displacements are ~~0.4055~~ 0.4025 in. for east-weat direction and 0.5127 in. for north-south direction respectively. These displacements are insignificant for a twenty-seven story building.

It is concluded that Academy Towers has sufficient resistance to earthquake load.

CHAPTER 6

DYNAMIC ANALYSIS

In dynamic analysis, often the period and the fundamental mode shape of the building is required. Two methods have been used to compute the fundamental mode shape and period of the building.

6-1 Rayleigh's Method

For East-West direction, Rayleigh's method is used. It is described as the following:

1. Apply a force to each floor. The force is equal to the dead weight of the floor.
2. Compute the deflections due to the forces applied. In this case, the programn STRUDL-II is again used to compute deflections. The deflections obtained show the fundamental mode shape of the building. Table 11 shows the fundamental mode shape of the building in East-West direction by Rayleigh's method.
3. By using the following formula, the period is computed.

$$U = \sqrt{g\frac{\Sigma\, Wi\ di}{\Sigma\, Wi\ di^2}}$$

$$T = \frac{2\pi}{U}$$

g = 32.2 ft/see ^{2}

Wi = weight of i^{th} floor

di = deflection of i^{th} floor

π = 3.1416

T = natural period

The period is 1.156 sec.

6-2 STRUDL-II

For north-south direction, the programn STRUDL-II is used. The programn would compute the mode shape and the period for several modes. (See Appendix A for the application of STRUDL-II). The natural period for the fundamental mode shape is 1.148 second. The fundamental mode shape is shown in Table 12.

CHAPTER 7

EFFECT OF STIFF BEAMS ON ROOF

The stiff beam on the roof is an unique feature. (For simplicity, the stiff beam has been transformed into T-beams for analysis. See Fig.) This idea has been proposed by Skilling, Heele, Christainin and Robertson, Structural engineer Richard Libbey applied such theory in practice.

7-1 Comparsion of Results

For comparsion, a similar analysis has been done, the same method is used, assuming that the stiff beams are absent. The table below shows the comparsion of the two analyses

	Direction	with Stiff beam	without Stiff beam
Maximum Deflection due to wind	east-west	0.5697 in.	0.9315 in.
	north-south	1.1281 in.	2.0153 in.
Maximum Deflection due to Earthquake	east-west	0.4025 in.	0.6684 in.
	north-south	0.5127 in.	0.9248 in.
Period of the building	east-west	1.156 sec.	1.489 sec.
	north-south	1.148 sec.	1.304 sec.

Table 3,5,7,9 show the deflections and Table 11,13 showthe fundamental mode shapes of the building assuming that the stiff beams on the roof are absent. Fig. 13,14, 15 and 16 show the comparsion of deflections.

CHAPTER 8

CONCLUSION, DISCUSSION AND RECOMMANDATION

The building consists of shear walls and frames. In this paper, instead of analysing the combined shear walls and frame (which is three dimensional), two plane frames (which are two dimensional) are analysed. The plane frame used are modified by treating the stiff shear walls as vertical members and slabs as horizontal members of the frames. (See Chapter 2 for more details). It is also assumed that the joints are perfectly rigid which is not true in reality. It should be noted that the above assumptions have been made in this paper.

8-1 Discussions and Recommandations for wind analysis

Wind load is treated as static loads instead of dynamic loads. Concentrated loads are used instead of uniform loading as suggested in Uniform Building Code. (See Chapter 4 for details). The computed concentrated loads are applied at the perfectly rigid joints of the modified plane frame. The deflections are computed with all the above assumptions. One should easily note that this is a simplified approach of analysis.

For a better analysis, all the assumptions should be reconsidered. A three dimensional frame combined with shear walls should be analysed instead of two modified plane frames. The rigidity of connections (between columns and slabs, shear walls and slabs) should be considered since they are not perfectly rigid. Uniform loads should be used instead of concentrated loads. The wind velocity on site should be used to calculate the wind load. The wind action on tall building should be investigated. Deflections measured from the building should be compared with the one obtained from analysis.

As far as engineering is concerned, a more detailed analysis will provide a more precise result, but this requires a lot more time and labor. If a rough estimate is required, the simplified analysis is good for sake of saving money and time. For a taller building such as one above fifty storeys, a more detailed analysis is highly recommanded because the assumptions might significantly affect the result.

The aim of design against wind is to minimize deflection. For large deflection, windows might break, walls might crack and failure might occur. For small deflection, it might lead to discomfort of residents. It comes to the question, "what should be the allowable deflection?" As suggested by Coull

$$d = \frac{h}{434} = 6.3 \text{ inches}$$

The results obtained are far below the suggested allowable value.

It is hard to decide what is the allowable deflection. Apart from the design criteria relating to the building and its components, the criteria of occupier comfort should also be considered.

8-2 Discussion and Recommandation for Earthquake Analysis

The analysis is based on the Uniform Building Code requirement. Again the computed loads are applied to the modified frames, thus the deflections are computed. In some other countries such as Canada they consider more variables.

From Uniform Building Code of U.S.:

$$V = ZKCW$$

From National Building Code of Canada:

$$V = \cancel{Z}ISKCW$$

I is importance factor. I = 1.0, 1.3.

For buildings like hospitals, houses for the disabled, I = 1.3,for most buildings I = 1.0.

S = 1.0, 1.5,subsoil factor.

This factor depends on soil properties.

For a better analysis, value for the variables used must be reconsidered. A dynamic analysis based on an appropriate earthquake motion for the site and the soil condition is even better than the analysis used. (Uniform Building Code assumes static loads for earthquake.)

Several factors that an earthquake engineer should know.

1. He should have an idea of the probability of occurence and size of earthquake. The ground motions of earthquake should be studied. It is found that earthquake motion is not only lateral but also vertical. In this analysis,

it is assumed that earthquake motion is only lateral.

2. The structural behavior should also be investigated. This can be done by installing recording devices in typical structural systems in all of the seismic areas and wait for the earthquake to come. Another method is to develop a large earthquake simulator which can produce ground motion and test a full scale building.

3. Besides understanding the earthquake and structural behavior, an earthquake engineer should also have some knowledge of the interaction between soil and the structure. There are basically two interaction effects between a tall building and the foundation soils.

   a. Physical interaction effects which involve the effects of stresses and deformations at the contact boundaries between structure and soil. Potential consequences of such effects include a change in ground response adjacent to the building, changes in period of the building or in deformations of the upper floors of the building resulting from rocking deformations of the underlying soil, and changes in response of the building due to soil deformations.

b. Response interaction, involving changes in response of a given type of structure as a result of changes in the response of different soil deposits to earthquake-induced motions in the underlying rock.

The aims of design against earthquake are for life protection and to minimize structural damage. The collapse of a building during earthquake means a lot of injuries or deaths. What we need is a building that won't collapse during earthquake. Ductility, which is measured by the area under the strain-stress curve, plays an important role. A ductile material means that it is able to absorb energy. If a building is ductile, it is able to absorb part of the energy released by the earthquake. There will be inelastic deformation but the main thing is to avoid collapse. This means a lot of injuries and lives are saved.

CONCLUSION

Since more and more tall buildings will be built and taller buildings will be expected. Designers should pay more attention to the design against wind load and earthquake load. For wind, research has tended to focus on the mean velocity profile; the recurrence of extreme wind speeds and the various turburlent properties of natural wind. Full-scale investigations of wind action on tall buildings gives the true behavior of a tall building. Development of wind tunnel testing gives us some precious information.

For earthquake, we lack of earthquake data to understand ground motion thoroughly. Generally, earthquake engineers do a elastic (or linear) dynamic analysis based on a recorded earthquake ground motion. Methods of inelastic dynamic analysis have be developed since most buildings behave inelastically during earthquake. Those methods need high-speed computer for the vast amount of computation involved. Besides all those points mentioned in this report, engineers should also look into the effect of repetitive loading. Through research, experiments, experience and failures, engineers learn how to design against seismic load.

REFERENCES

1. 1973 Uniform Building Code
2. 1971 Building Code Requirement for Reinforced Conrete (ACI 318-71)
3. Design of Steel Structures, 2nd edition, by Edwin H. Gaylord, Jr. and Charles N. Gaylord.
4. Tall Buildings, by Coull and Stafford Smith
5. Wind Loads on Structures, by Davenport, A. G.
6. Response Spectra of Free- Standing Towers subjected to Time-Varying Wind Forces, thesis for master degree, by David K. Watanabe.

APPENDIX A

THE APPLICATION OF STRUDL-II

STRUDL stands for structural design langage. It was developed by Massachusetts Institute of Technology, Civil Engineering Systems Laboratory.

The following are steps of application of STRUDL:

1. Specify the type of analysis. In this case, put down ' TYPE PLANE FRAME '.
2. Name each joint and member by assigning a specific number to each one of them, for example, a for a joint and m for a member.
3. Specify each joint's location such as (a x,y) where a is the number for the joint ; x is the horizontal distance and y is the vertical distance from a fixed coorindate. Put all those under the heading 'JOINT COORDINDATE'.
4. Specify each member by reading in its connection to the joints such as (m a b) where m is the number for the member; a and b are the numbers for the joints.

Put all those under the heading 'MEMBER INCIDENCES'.

6. Specify the modulus of elasticity of each member such as (CONSTANT E   m). Fill in the blank with the modulus of elasticity of the member m.

7. For computing deflections due to lateral loadings, specify the loading at each joint such as (JOINT a LOAD FORCE X f) where _a_ is the number for the joint and _f_ is the lateral force. Put all these loadings under the heading 'LOADING 1 'HORITZONTAL' '. After putting down all the loadings , put down ' LOADING LIST ALL ' , ' STIFFNESS ANALYSIS ', ' LIST LOAD,FORCES,REACTIONS,DISPLACEMENT,ALL ACTIVE JOINTS AND MEMBERS ' in three different lines.

FIG. 1

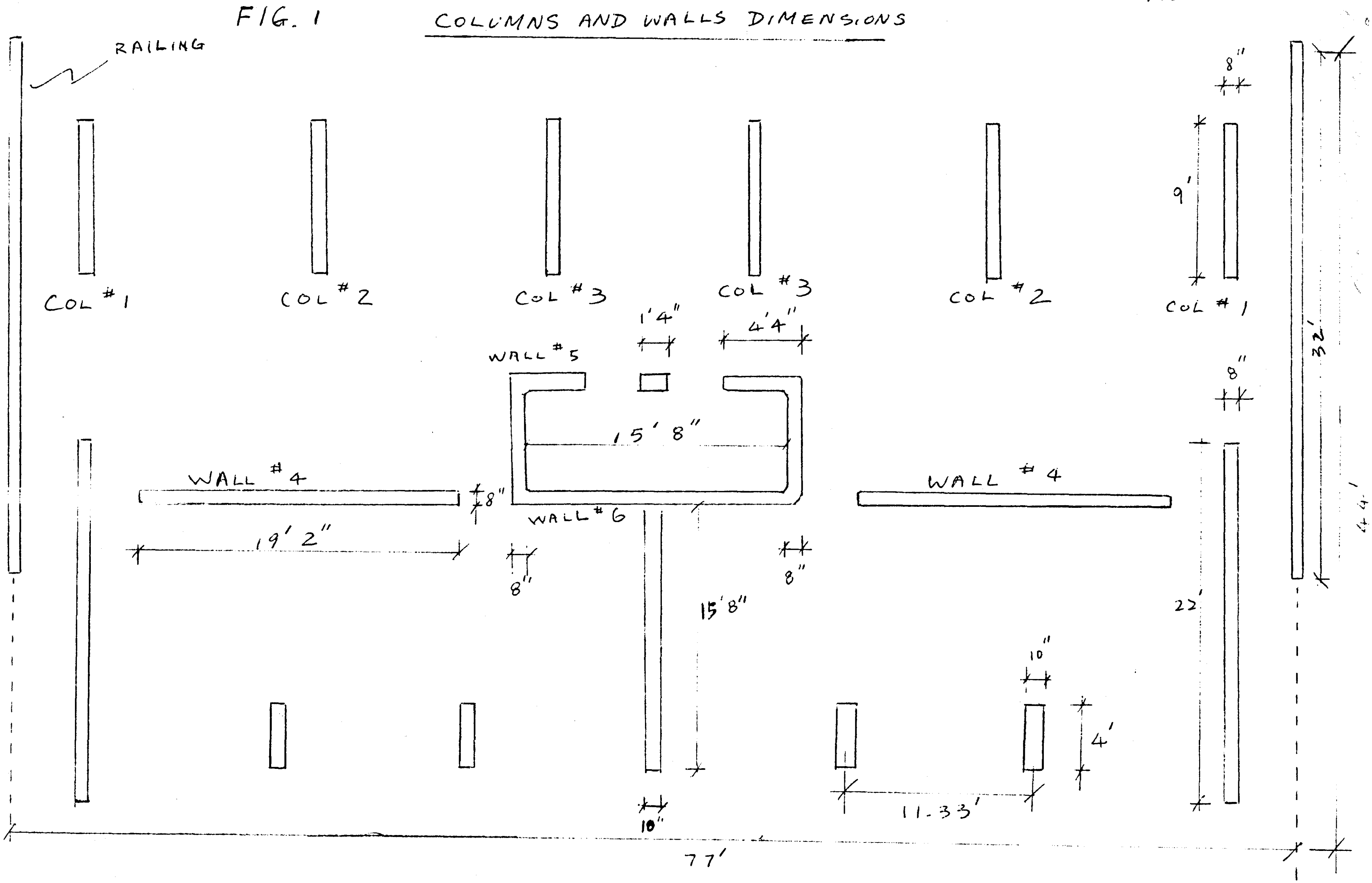

COLUMNS AND WALLS DIMENSIONS
RAILING
COL #1
COL #2
COL #3
COL #3
COL #2
COL #1
8"
9'
1'4"
4'4"
WALL #5
8"
15' 8"
WALL #4
WALL #4
8"
WALL #6
19' 2"
8"
8"
15'8"
22'
10"
4'
10"
11.33'
77'
32'
44'

FIG. 2

LOCATION OF CENTROID

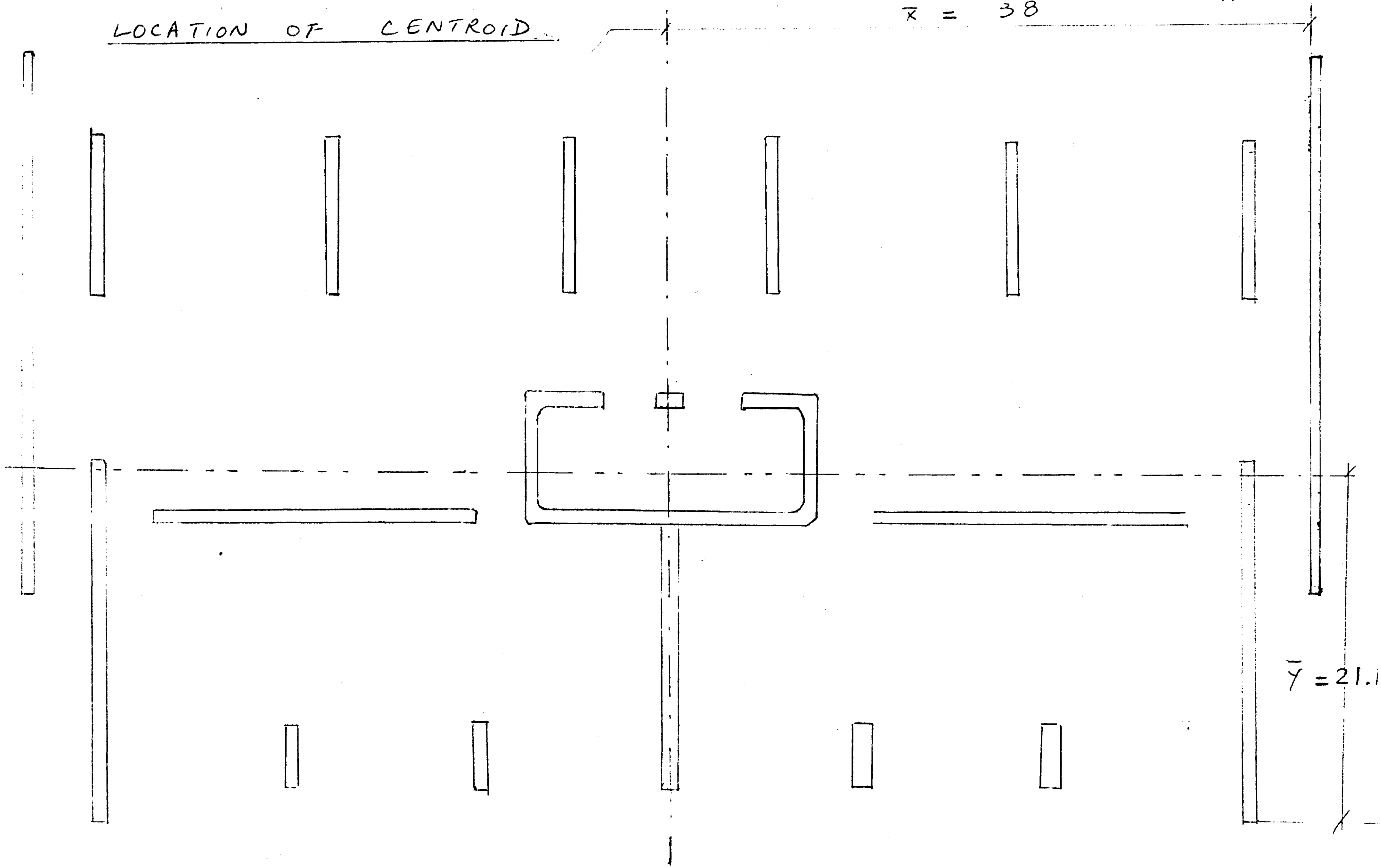
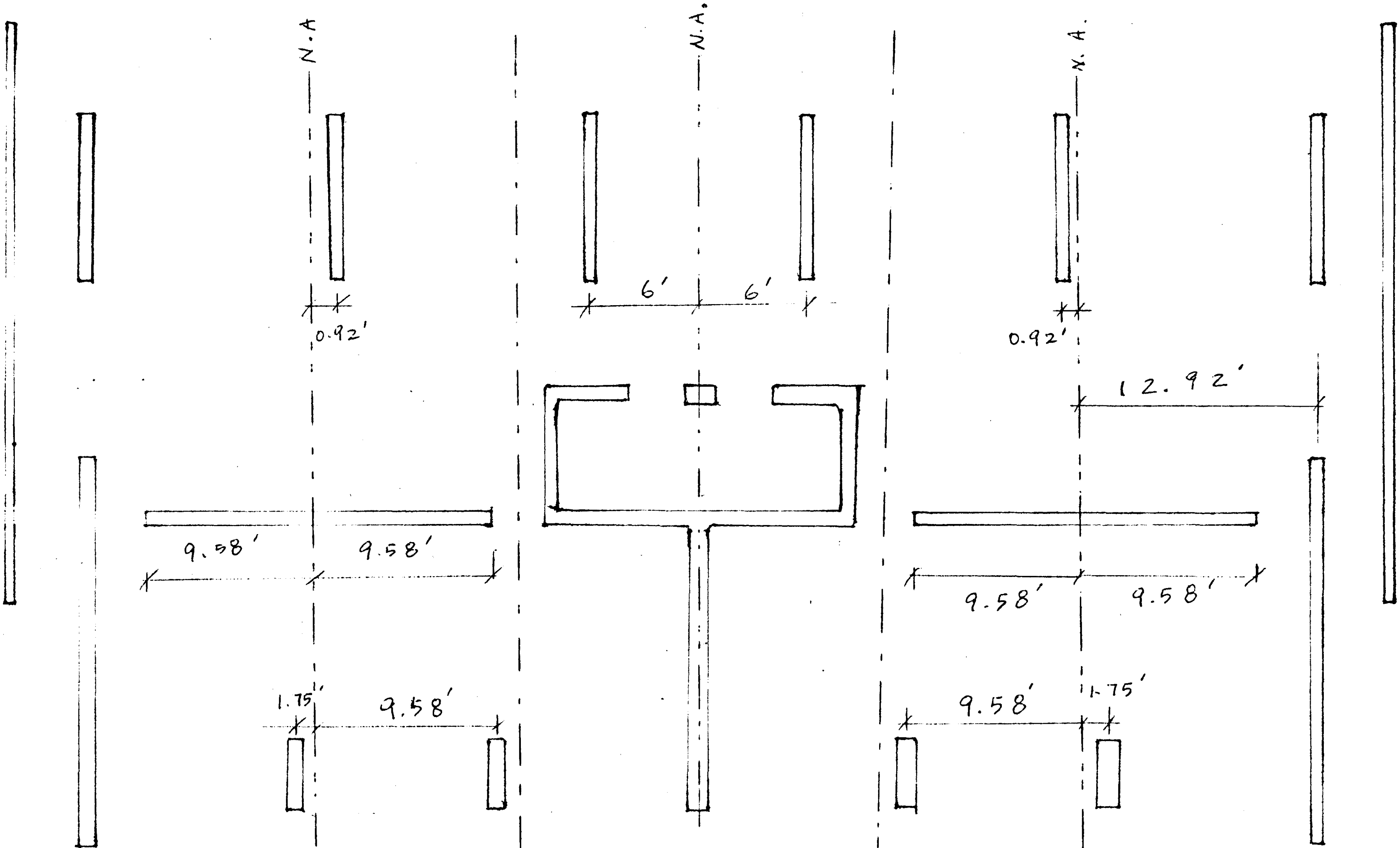


FIG. 3

SUBDIVISIONS OF THREE FRAMES



AREA = 46.11 ft^2

I_{yy} = 4195.38 ft^4

BSMT TO 4TH FLOOR

AREA = 54.2 ft^2

I_{yy} = 1572.12 ft^4

5TH FLOOR TO ROOF

AREA = 51.2 ft^2

I_{yy} = 1264.12 ft^4

AREA = 46.11 ft^2

I_{yy} = 4195.38 ft^4

FIG 4

SUBDIVISION INTO TWO FRAMES

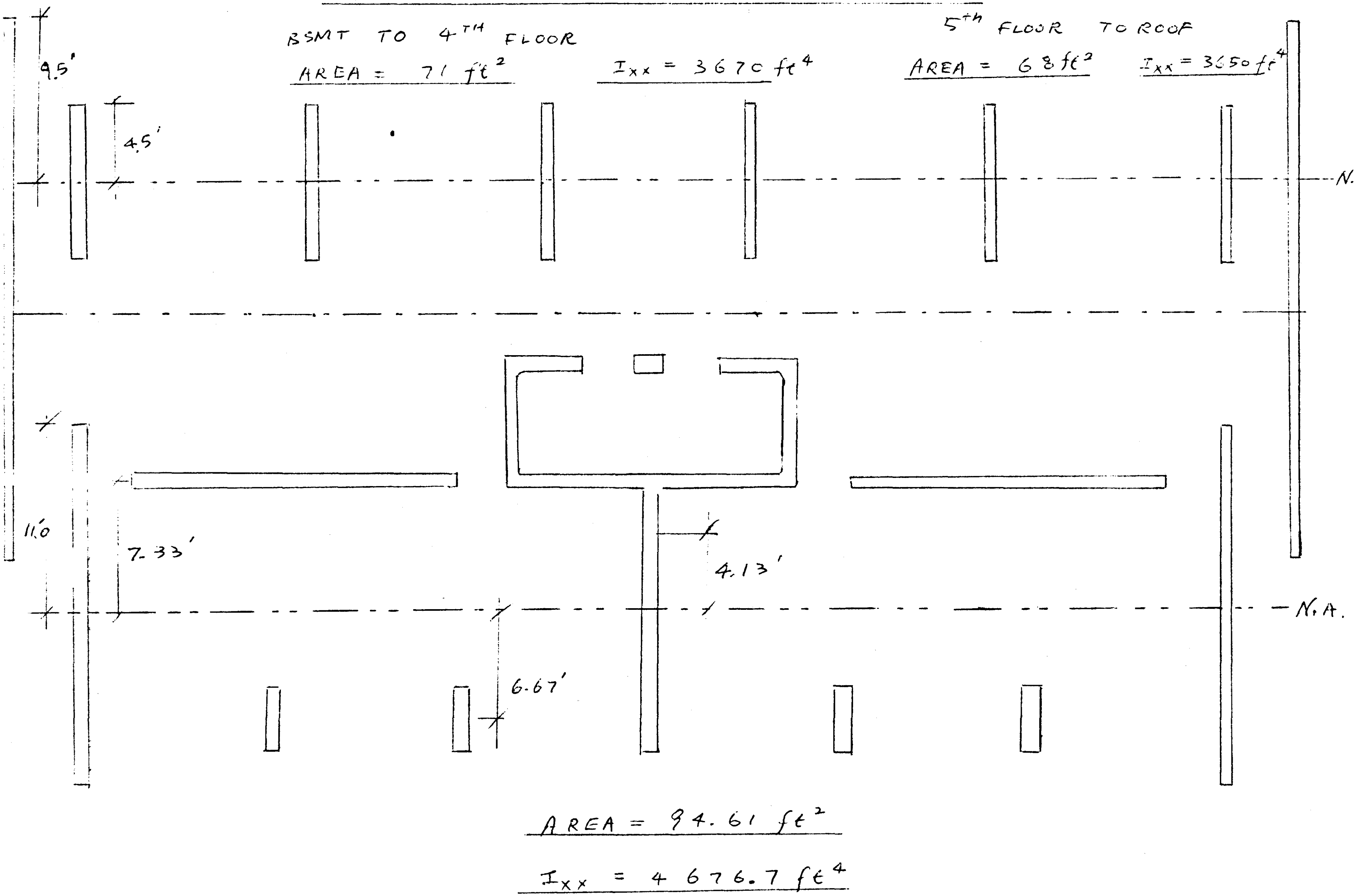
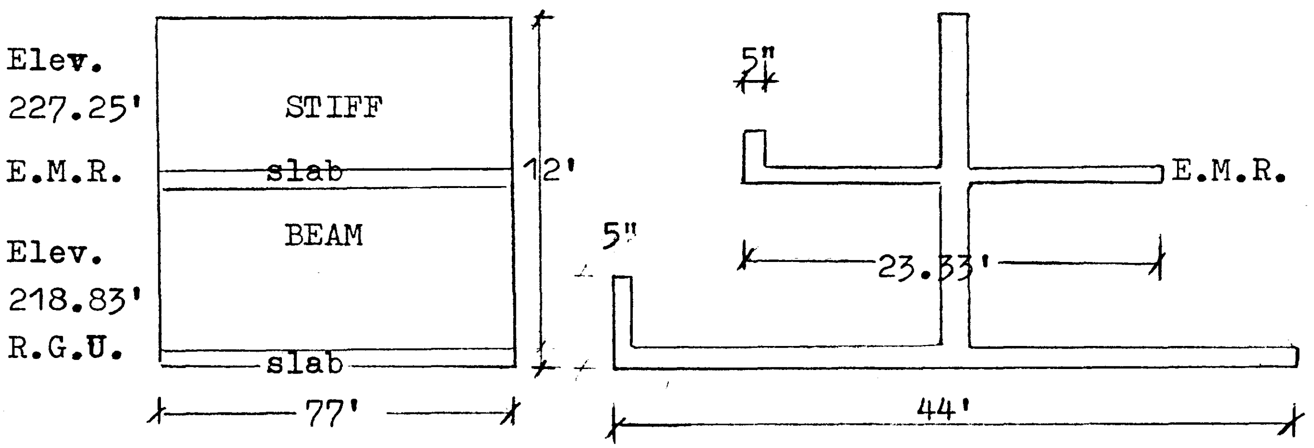


Fig. 5

ANALYSIS OF STIFF BEAM AS EQUIVALENT T-BEAMS

For East-West direction:



TRANFORMED T-SECTION

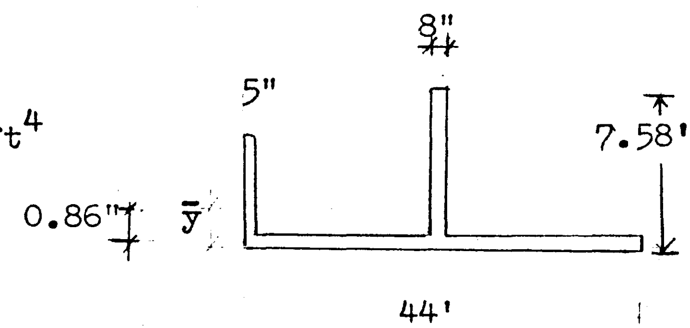
R.G.U. : $\bar{y}$ = 1.07'

area = 23.38 ft^2 I_{xx} = 87.79 ft^4

For JOINT 79,80,81 :

Elevation = 218.83' + 0.86'

= 219.69'



MEMBER 53,54

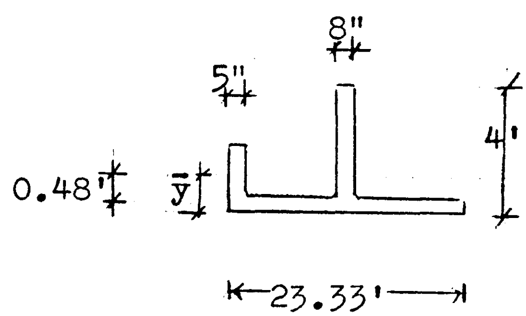
E.M.R. : $\bar{y}$ = 0.48'

area = 12.39 ft^2 I_{xx} = 13.9 ft^4

For JOINT 82,83,84 :

Elevation = 227.25'+ 0.48'

= 227.73'

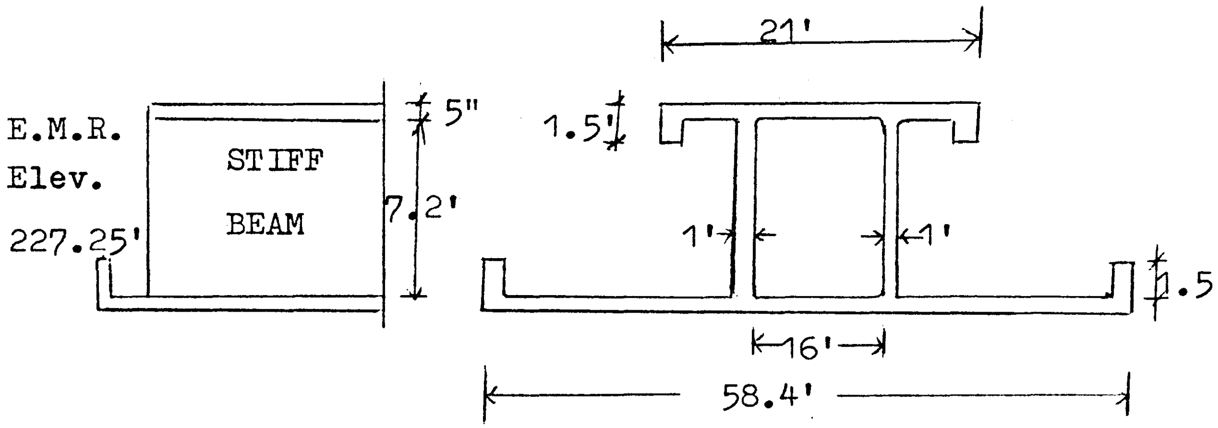


MEMBER 55,56

FIGURE 6

ANALYSIS OF STIFF BEAM AS EQUIVALENT DOUBLE-T-BEAM

For North-South direction :



MEMBER 28

E.M.R. :

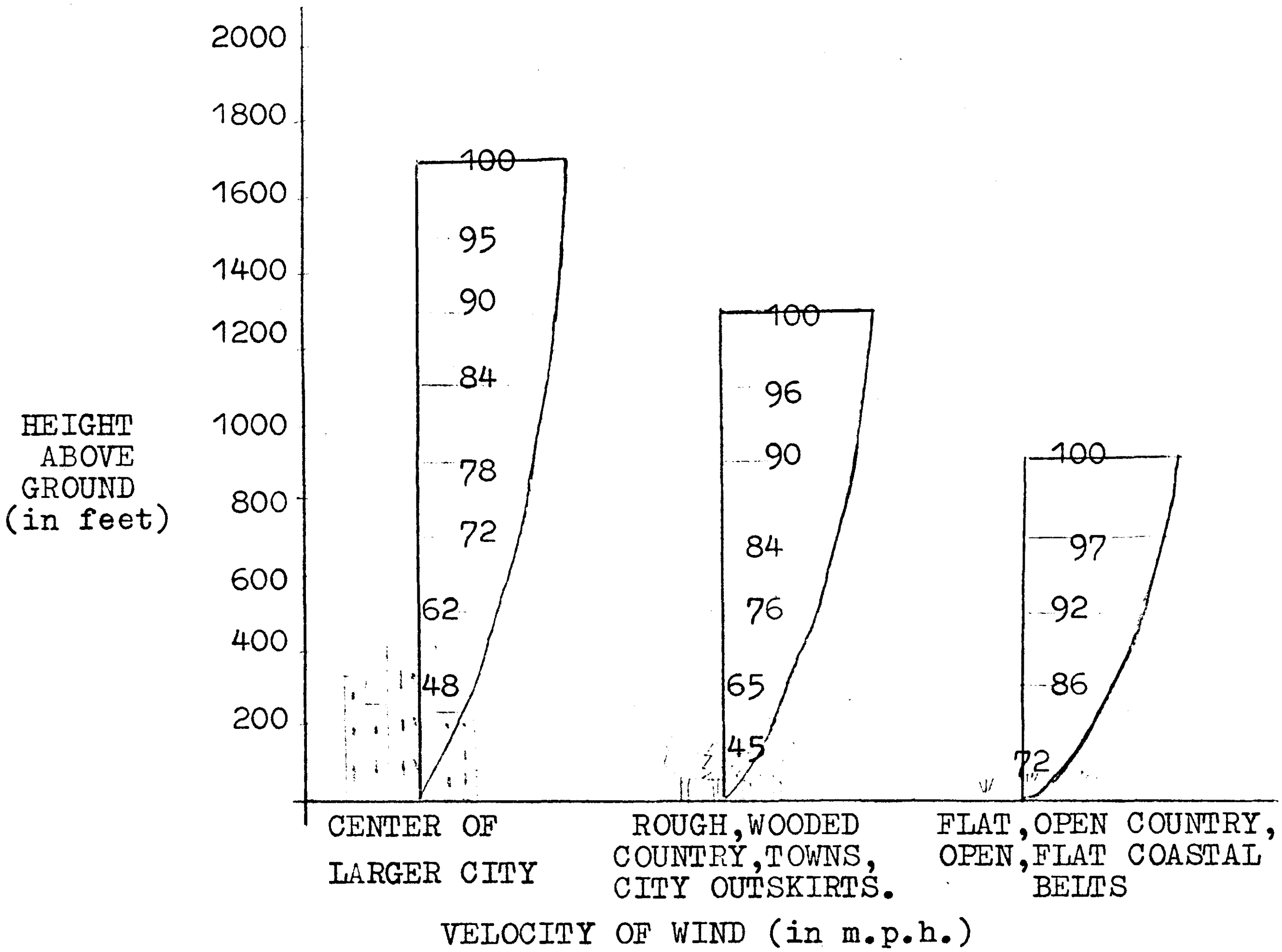
area = 63.07 ft^2

I_{xx} = 847.73 ft^4

For JOINT 55,56 : elevation is 231.06'

FIGURE 7

WIND VELOCITY PROFILES



VELOCITY PROFILES OVER TERRAIN WITH THREE DIFFERENT ROUGHNESS CHARACTERISTICS FOR UNIFORM-GRADIENT WIND VELOCITY OF 100 nph.

SOURCE : A G. DAVENPORT: WIND LOADS ON STRUCTURES

FIGURE 8

WIND LOAD DISTRIBUTION

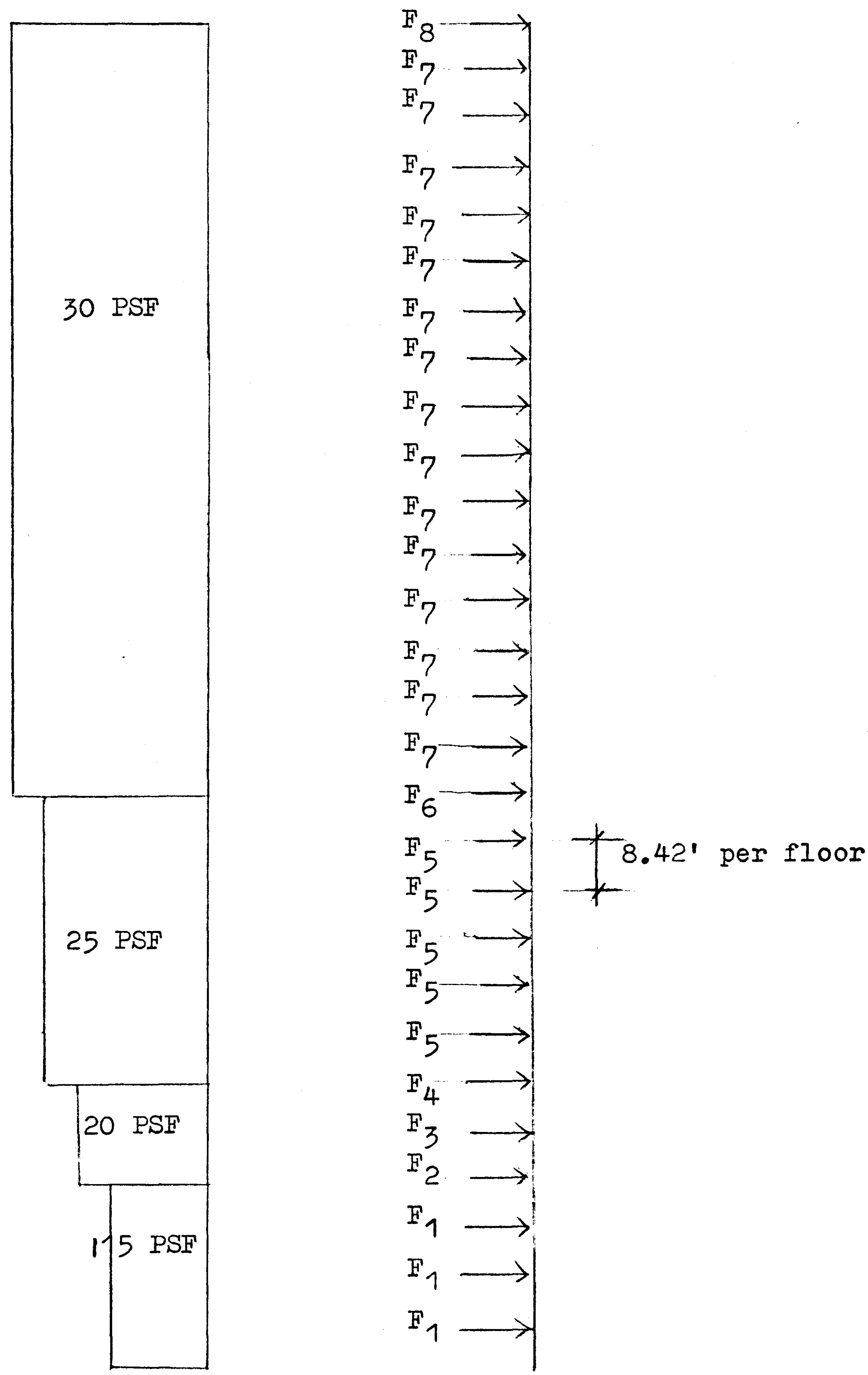


FIGURE 9

WIND ANALYSIS

FOR EAST-WEST DIRECTION

$F_1 = 15 \times 77' \times 8.42 = 9725\,\# = \underline{9.73}^{K}$

$F_2 = (15 \times 77' \times 0.53') + (20 \times 77' \times 7.89') = \underline{12.76}^{K}$

$F_3 = 20 \times 77' \times 8.42' = \underline{12.97}^{K}$

$F_4 = (20 \times 77' \times 3.89') + (25 \times 77' \times 4.53') = \underline{14.71}^{K}$

$F_5 = 25 \times 77' \times 8.42' = \underline{16.21}^{K}$

$F_6 = (25 \times 77' \times 3.17') + (30 \times 77' \times 5.25') = \underline{18.23}^{K}$

$F_7 = 30 \times 77' \times 8.42' = \underline{19450}^{K}$

$F_8 = 0.5 \times F_7 = \underline{9.73}^{K}$

FOR NORTH-SOUTH DIRECTION

$F_1 = 15 \times 44' \times 8.42' = \underline{5.56}^{K}$

$F_2 = (15 \times 44' \times 0.53) + (20 \times 44' \times 7.89') = \underline{7.29}^{K}$

$F_3 = 20 \times 44' \times 8.42' = \underline{7.41}^{K}$

$F_4 = (20 \times 44' \times 3.89') + (25 \times 44' \times 4.53') = \underline{8.41}^{K}$

$F_5 = 25 \times 44' \times 8.42' = \underline{9.26}^{K}$

$F_6 = (25 \times 44' \times 3.17') + (30 \times 44' \times 5.25') = \underline{10.42}^{K}$

$F_7 = 30 \times 44' \times 8.42 = \underline{11.11}^{K}$

$F_8 = 0.5 \times F_7 = \underline{5.56}^{K}$

FIGURE 10

EARTHQUAKE ANALYSIS

FOR EAST-WEST DIRECTION

$V = ZKCW \qquad Z = 0.25$

$K = 1$

$C = \frac{0.05}{(T)^{1/3}} = 0.0464 \quad \left(T = \frac{0.05hn}{\sqrt{D}} \right)$

$W = 11{,}200^K$

$V = 129.87^K$

$F = 0.004\ V \left(\frac{hn}{D_s}\right)^2 = 4.197^K$

$$F = \frac{(V - F_t)\ W_x h_x}{\sum_{i=1}^{n} Wihi}$$

FOR NORTH-SOUTH DIRECTION

$V = ZKCW \qquad Z = 0.25$

$K = 1$

$C = \frac{0.05}{T^{1/3}} = 0.0423 \quad \left(T = \frac{0.05hn}{\sqrt{D}} \right)$

$W = 11{,}200^K$

$V = 118.44^K$

$F = 0.004\ V \left(\frac{hn}{D_s}\right)^2 = 11.72^K$

$$F = \frac{(V - Ft)\ Wxhx}{\sum_{i=1}^{n} Wihi}$$

FIGURE 11

EARTHQUAKE LOAD DISTRIBUTION

24.98^K	13.63^K	→ TOP
10.38^K	10.17^K	→ 27
9.99^K	9.78^K	→ 26
9.59^K	9.39^K	→ 25
9.19^K	9.00^K	→ 24
8.79^K	8.61^K	→ 23
8.39^K	8.22^K	→ 22
7.99^K	7.83^K	→ 21
7.59^K	7.44^K	→ 20
7.20^K	7.05^K	→ 19
6.80^K	6.66^K	→ 18
6.40^K	6.27^K	→ 17
6.00^K	5.88^K	→ 16
5.60^K	5.49^K	→ 15
5.20^K	5.10^K	→ 14
4.80^K	4.71^K	→ 13
4.41^K	4.32^K	→ 12
4.00^K	3.92^K	→ 11
3.61^K	3.54^K	→ 10
3.21^K	3.15^K	→ 9
2.81^K	2.75^K	→ 8
2.41^K	2.37^K	→ 7
2.01^K	1.97^K	→ 6
1.62^K	1.58^K	→ 5
1.22^K	1.19^K	→ 4
0.82^K	0.80^K	→ 3
0.42^K	0.43^K	→ 2
0.00^K	0.0[illegible]^K	→ 1

* TOP DEAD LOAD
* FROM TOP TO MACHINE ROOM IS 227^K

DEAD LOAD IS 400^K/FLOOR

* DEAD LOAD IS 408.6^K/FLOOR.

✓

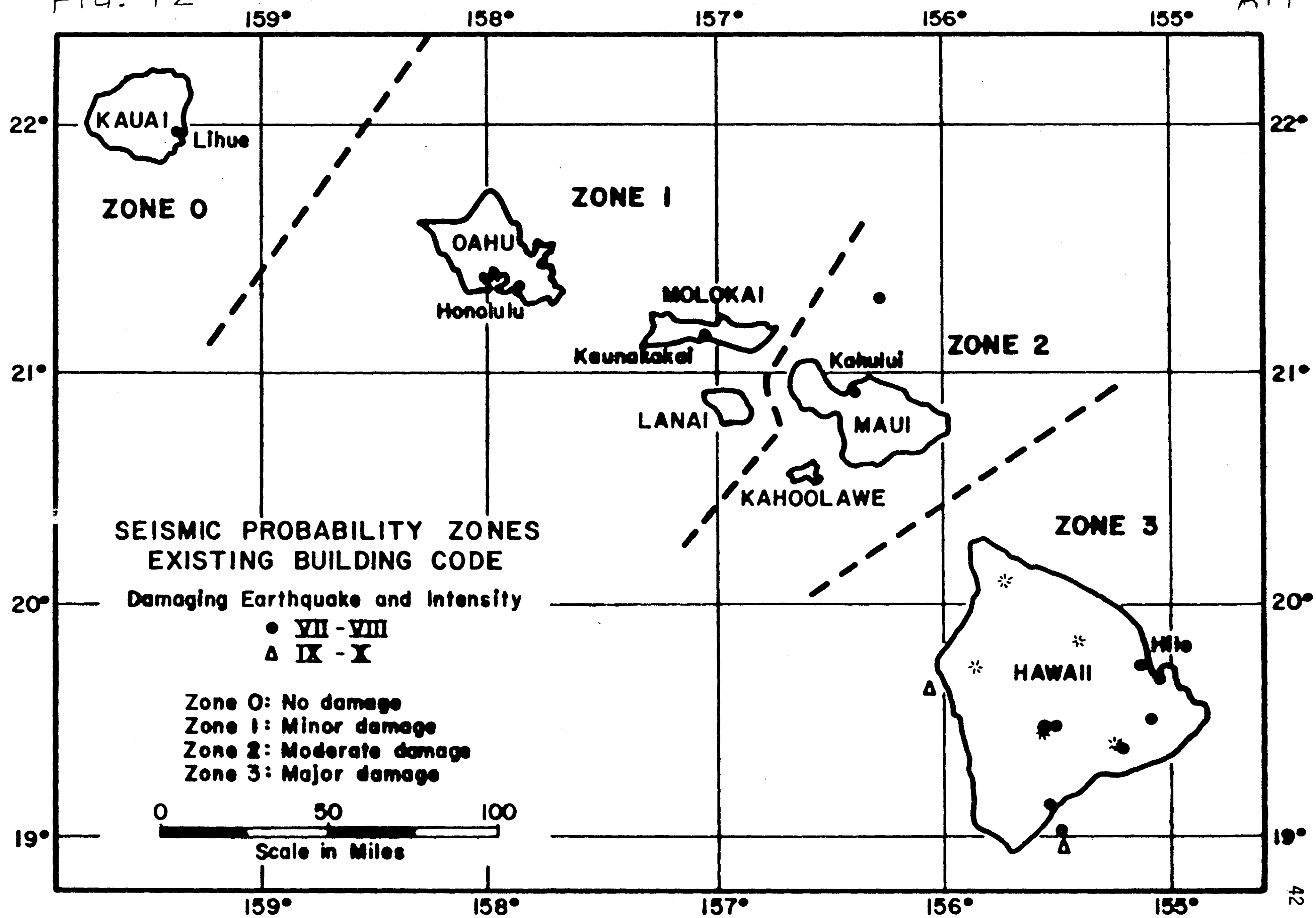

159°
158°
157°
156°
155°
22°
21°
20°
19°
KAUAI
Lihue
ZONE 0
ZONE 1
OAHU
Honolulu
MOLOKAI
Kaunakakai
ZONE 2
Kahului
LANAI
MAUI
KAHOOLAWE
ZONE 3
HAWAII
Hilo
SEISMIC PROBABILITY ZONES
EXISTING BUILDING CODE
Damaging Earthquake and Intensity
● VII - VIII
Δ IX - X
Zone 0: No damage
Zone 1: Minor damage
Zone 2: Moderate damage
Zone 3: Major damage
0
50
100
Scale in Miles

FIGURE 7.

FIGURE 13

DEFLECTIONS DUE TO WIND LOAD IN EAST-WEST DIRECTION :

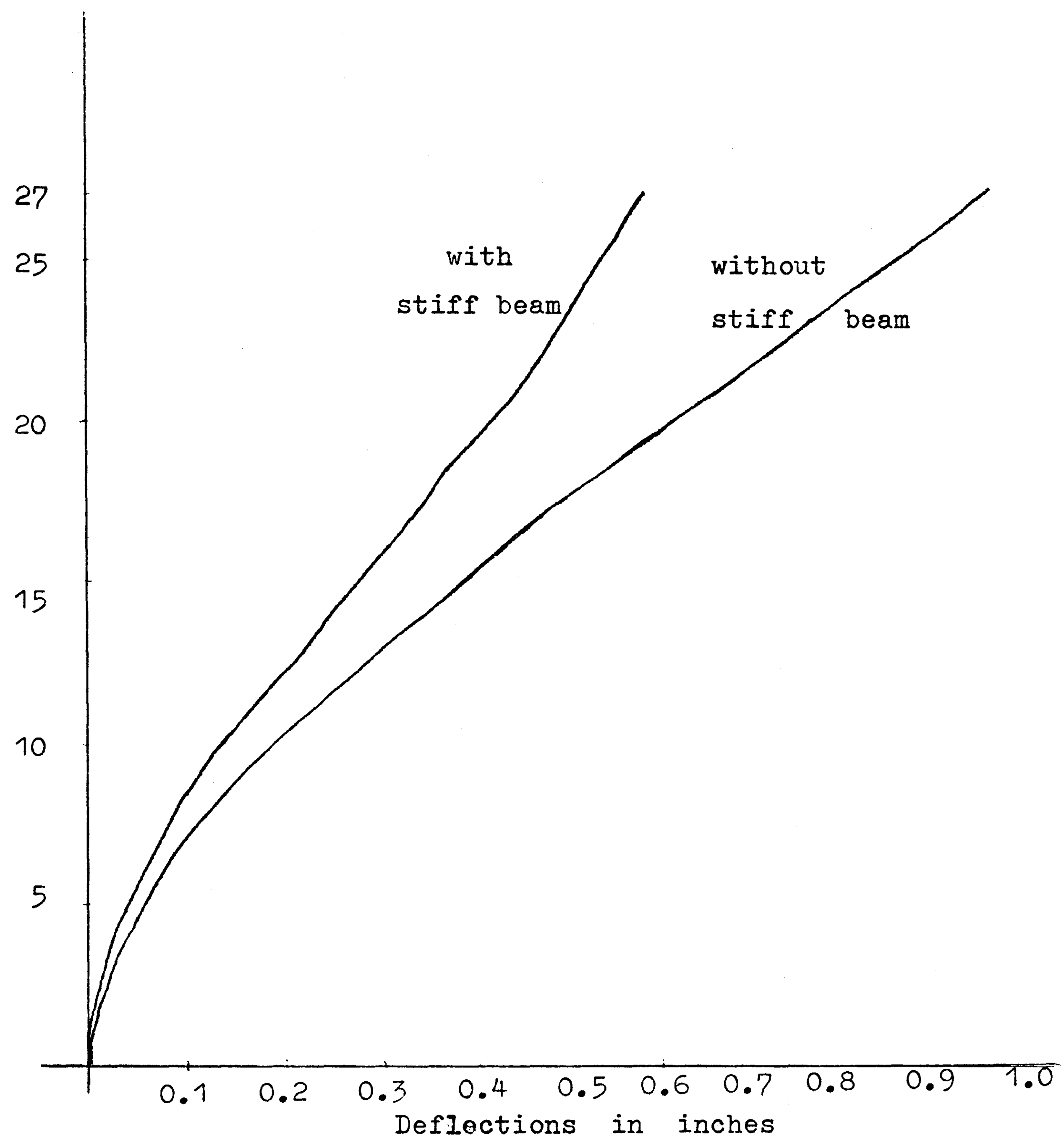


FIGURE 14

DEFLECTIONS DUE TO WIND LOAD IN NORTH-SOUTH DIRECTION :

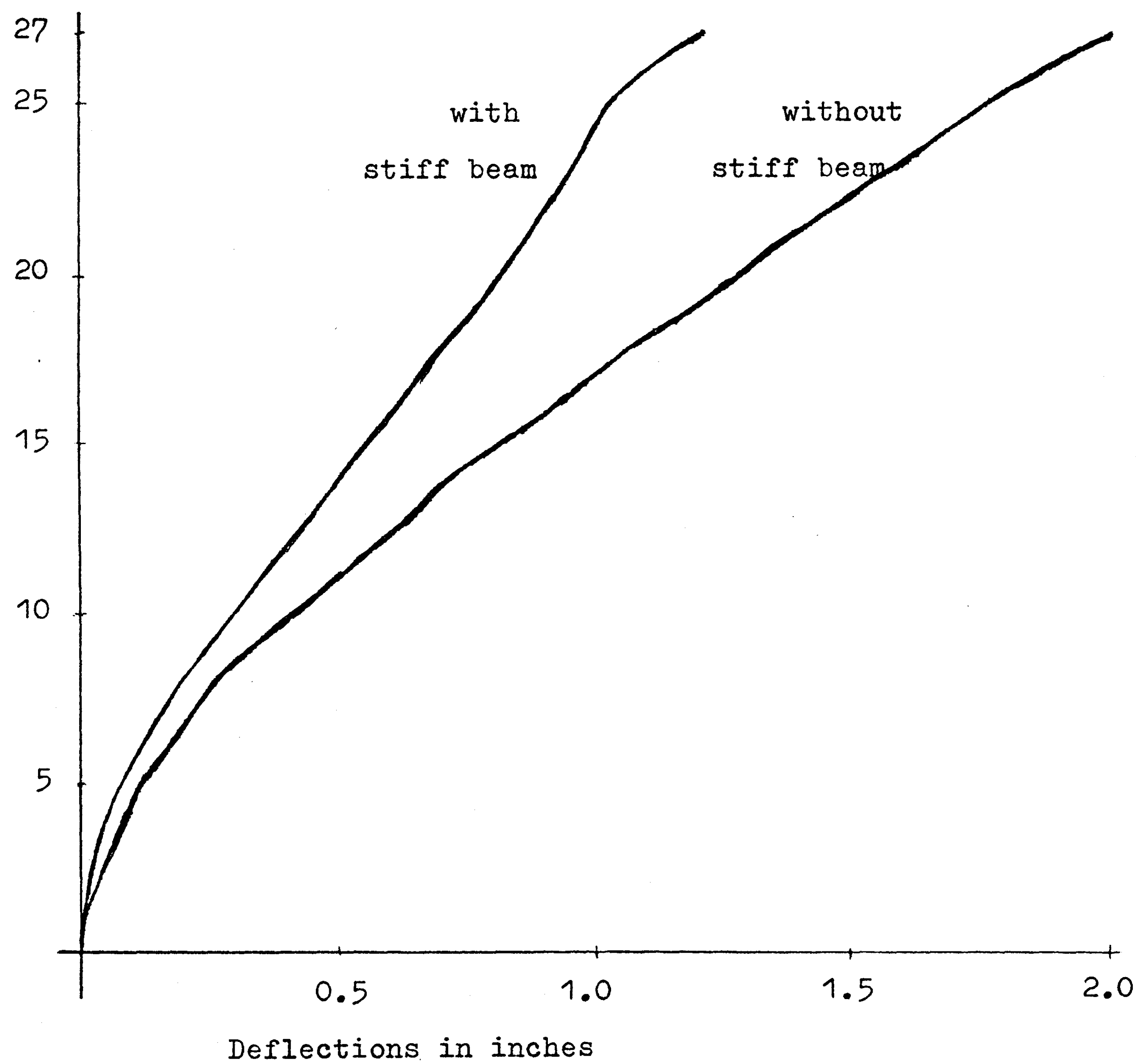


FIGURE 15

DEFLECTIONS DUE TO EARTHQUAKE LOAD

IN EAST-WEST DIRECTION :

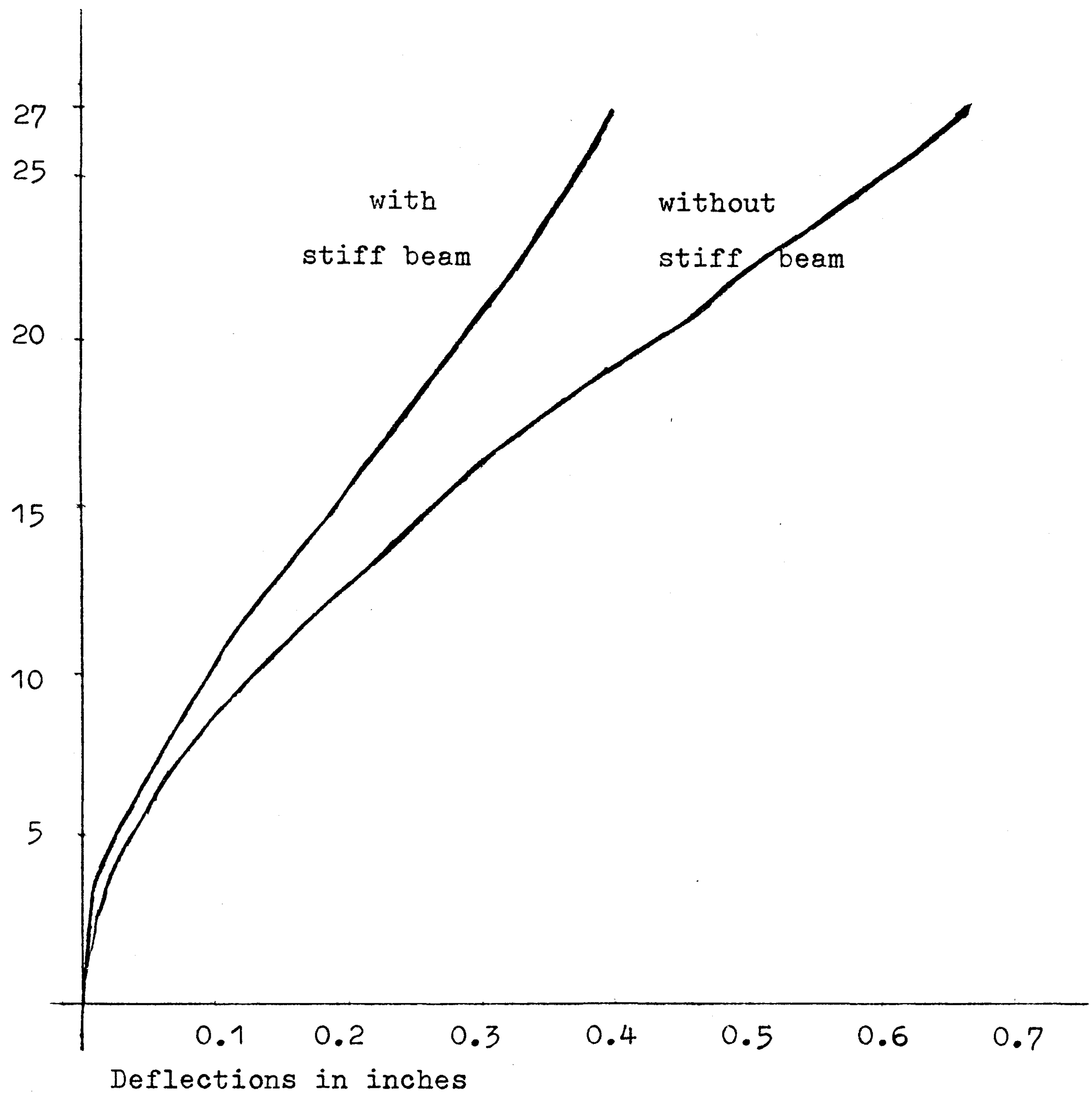


FIGURE 16

DEFLECTIONS DUE TO EARTHQUAKE LOAD

IN NORTH-SOUTH DIRECTION :

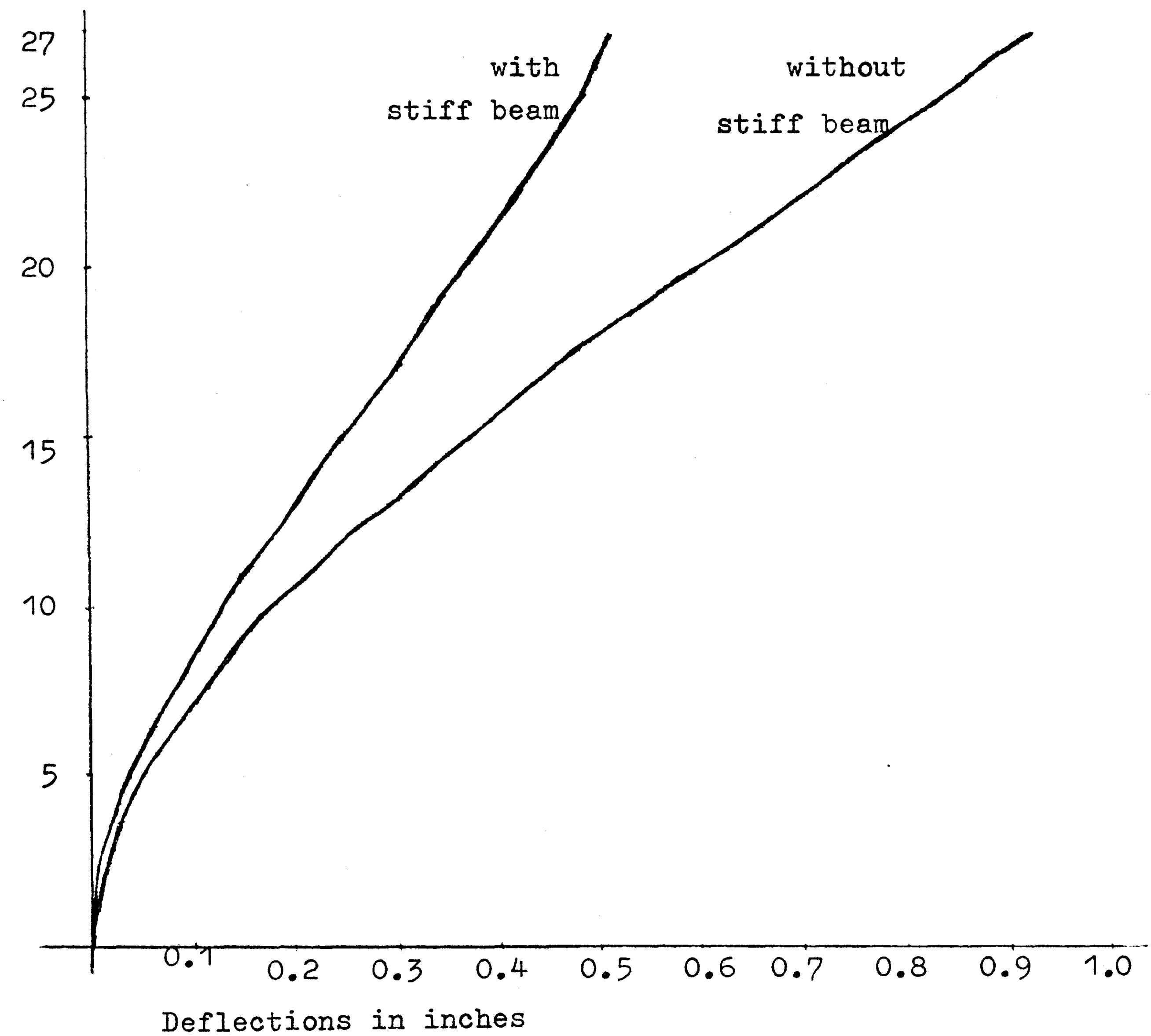


FIGURE 17

JOINTS AND MEMBERS FOR NORTH-SOUTH DIRECTION

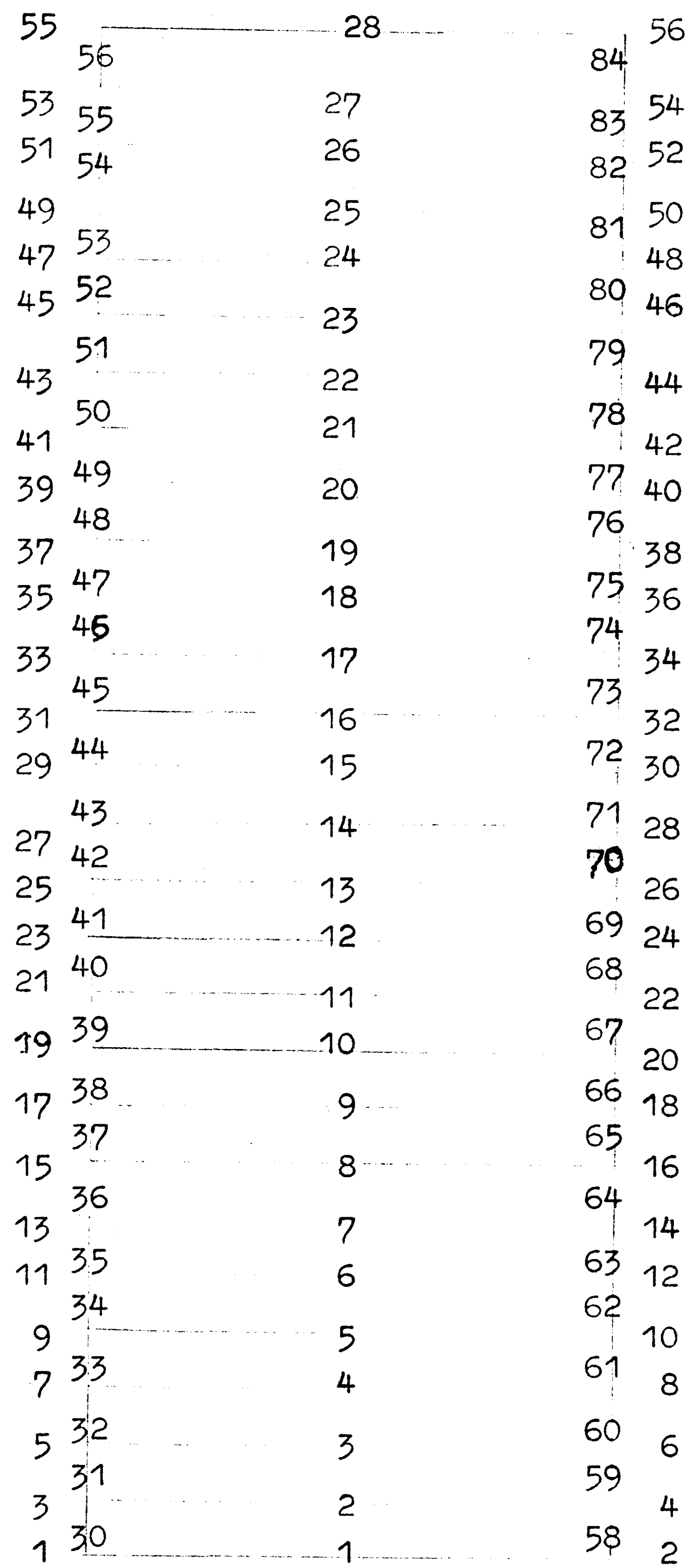


FIGURE 18

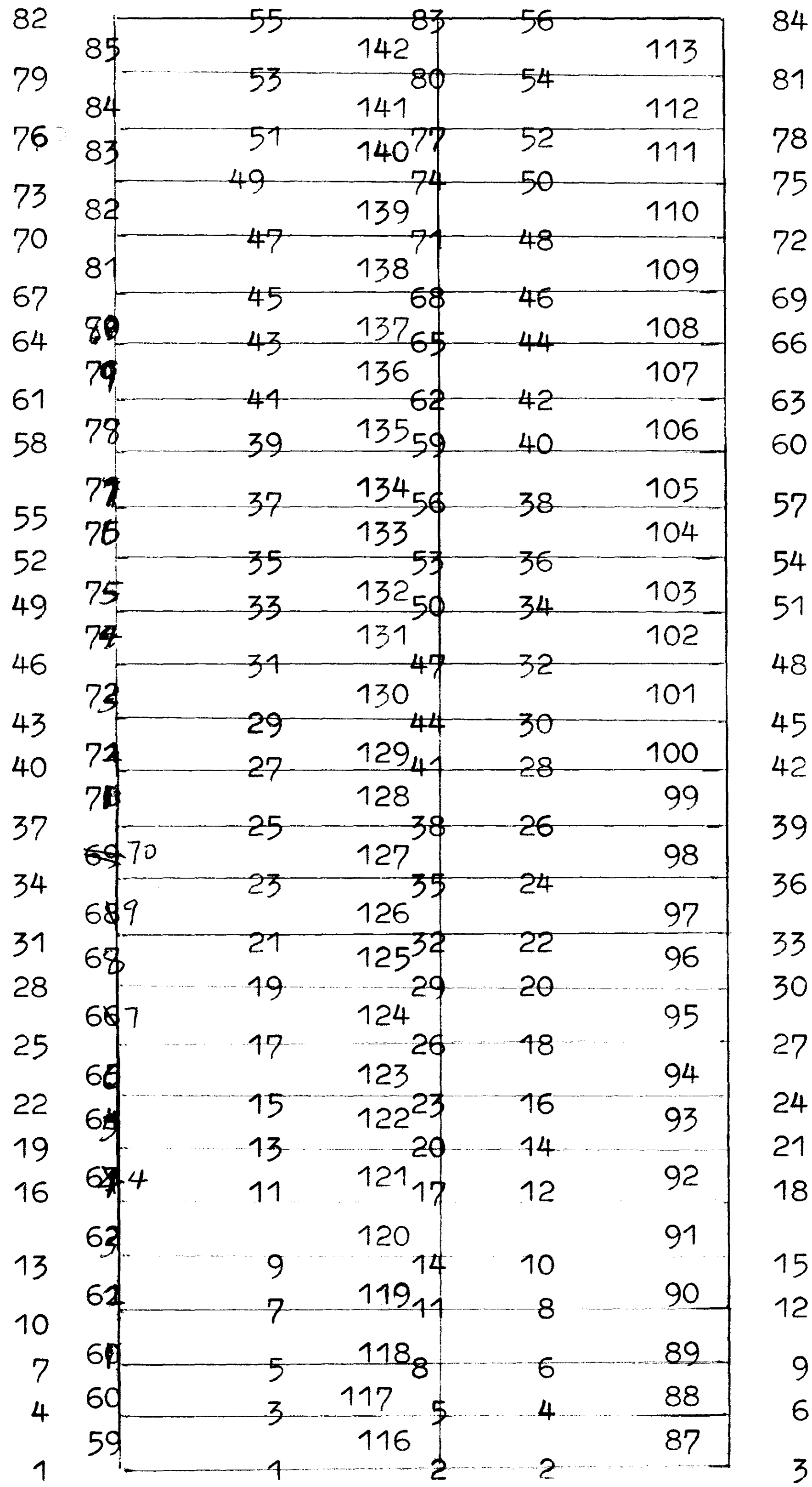

82 55 83 56 84
85 142 113
79 53 80 54 81
84 141 112
76 83 51 140 77 52 111 78
73 49 74 50 75
82 139 110
70 47 71 48 72
81 138 109
67 45 68 46 69
64 80 43 137 65 44 108 66
79 136 107
61 41 62 42 63
58 78 39 135 59 40 106 60
77 134 105
55 37 56 38 57
76 133 104
52 35 53 36 54
49 75 33 132 50 34 103 51
74 131 102
46 31 47 32 48
73 130 101
43 29 44 30 45
40 72 27 129 41 28 100 42
71 128 99
37 25 38 26 39
69 70 127 98
34 23 35 24 36
68 69 126 97
31 21 125 32 22 96 33
68
28 19 29 20 30
66 67 124 95
25 17 26 18 27
65 123 94
22 15 122 23 16 93 24
64
19 13 20 14 21
16 63 64 11 121 17 12 92 18
62 120 91
13 9 14 10 15
61 119 11 7 8 90 12
10
7 60 5 118 8 6 89 9
4 60 3 117 5 4 88 6
59 116 87
1 1 2 2 3

TABLE 1

AREAS AND MOMENTS OF INERTIA OF COLUMNS AND WALLS

	AREA (ft^2)	I (ft^4)	I (ft^4)
COL # 1	6	40.5	0.22
COL # 2	6	40.5	0.22
COL # 3* (BSMT To 4 FLR)	7.5	50.63	0.43
WALL # 4	12.78	0.47	422.6
WALL # 5 # 6 # 10	39.17	1,529.64	1,032.12
WALL # 7	14.67	591.56	0.54
COL # 8	3.33	4.44	0.19
COL # 9	3.33	4.44	0.19
COL # 3 (5 FLR TO ROOF)	6	40.5	0.22

*

TABLE 2

DEFLECTIONS DUE TO WIND LOAD IN EAST-WEST DIRECTION (WITH STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENTS (IN INCHES)
1	8.42	0.0018
2	16.83	0.0071
3	25.25	0.0155
4	33.67	0.0268
5	42.08	0.0408
6	50.50	0.0574
7	58.90	0.0762
8	67.33	0.0969
9	75.75	0.1193
10	84.16	0.1432
11	92.58	0.1682
12	101.00	0.1943
13	109.42	0.2210
14	117.83	0.2482
15	126.25	0.2756
16	134.66	0.3031
17	143.08	0.3305
18	151.50	0.3576
19	159.92	0.3844
20	168.33	0.4103
21	176.75	0.4357
22	186.16	0.4603
23	193.58	0.4841
24	202.00	0.5070
25	210.46	0.5288
26	219.69	0.5505
27	227.73	0.5697

TABLE 3

DEFLECTIONS DUE TO WIND LOAD IN EAST-WEST DIRECTION (WITHOUT STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0023
2	16.83	0.0089
3	25.25	0.0195
4	33.67	0.0339
5	42.08	0.0519
6	50.50	0.0735
7	58.90	0.0983
8	67.33	0.1261
9	75.75	0.1565
10	84.16	0.1893
11	92.58	0.2245
12	101.00	0.2617
13	109.42	0.3006
14	117.83	0.3410
15	126.25	0.3827
16	134.66	0.4256
17	143.08	0.4694
18	151.50	0.5140
19	159.92	0.5593
20	168.33	0.6050
21	176.75	0.6512
22	185.16	0.6975
23	193.58	0.7442
24	202.00	0.7910
25	210.46	0.8380
26	219.69	0.8846
27	227.73	0.9315

TABLE 4

DISPLACEMENT DUE TO WIND LOAD IN NORTH-SOUTH DIRECTION (WITH STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0004
2	16.83	0.0145
3	25.25	0.0316
4	33.67	0.0545
5	42.08	0.0827
6	50.50	0.1161
7	58.90	0.1540
8	67.33	0.1957
9	75.75	0.2407
10	84.16	0.2885
11	92.58	0.3385
12	101.00	0.3902
13	109.42	0.4431
14	117.83	0.4967
15	126.25	0.5506
16	134.66	0.6042
17	143.08	0.6575
18	151.50	0.7098
19	159.92	0.7609
20	168.33	0.8105
21.	176.75	0.8584
22	186.16	0.9042
23	193.58	0.9480
24	202.00	0.9893
25	210.00	1.0282
26	219.73	1.0645
27	227.73	1.12814

TABLE 5

DEFLECTION DUE TO WIND LOAD IN NORTH-SOUTH DIRECTION (WITHOUT STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0048
2	16.83	0.0187
3	25.25	0.0412
4	33.67	0.0715
5	42.08	0.1045
6	50.50	0.1548
7	58.90	0.2073
8	67.33	0.2659
9	75.75	0.3302
10	84.16	0.3978
11	92.58	0.4741
12	101.00	0.5527
13	109.42	0.6349
14	117.83	0.7203
15	126.25	0.8086
16	134.66	0.8991
17	143.08	0.9918
18	151.50	1.0861
19	159.92	1.1819
20	168.33	1.2786
21	176.75	1.3763
22	185.16	1.4745
23	193.58	1.5732
24	202.00	1.6723
25	210.46	1.7716
26	219.69	1.8709
27	227.73	2.0153

TABLE 6

DEFLECTIONS DUE TO EARTHQUAKE LOAD IN EAST-WEST DIRECTION (WITH STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0012
2	16.83	0.0047
3	25.25	0.0103
4	33.67	0.0178
5	42.08	0.0272
6	50.50	0.0384
7	58.90	0.0511
8	67.33	0.0653
9	75.75	0.0807
10	84.16	0.0972
11	92.58	0.0115
12	101.00	0.1329
13	109.42	0.1517
14	117.83	0.1709
15	126.25	0.1905
16	134.66	0.2101
17	143.08	0.2297
18	151.50	0.2492
19	159.92	0.2685
20	168.33	0.2874
21	176.75	0.3058
22	185.16	0.3238
23	193.58	0.3411
24	202.00	0.3579
25	210.46	0.3739
26	219.69	0.3897
27	227.73	0.4025

(6)

TABLE 7

DEFLECTION DUE TO EARTHQUAKE LOAD IN EAST-WEST DIRECTION (WITHOUT STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0015
2	16.83	0.0060
3	25.25	0.0132
4	33.67	0.0230
5	42.08	0.0353
6	50.50	0.0501
7	58.90	0.0673
8	67.33	0.0866
9	75.75	0.1079
10	84.16	0.1310
11	92.58	0.1558
12	101.00	0.1822
13	109.42	0.2099
14	117.83	0.2388
15	126.25	0.2688
16	134.66	0.2997
17	143.08	0.3314
18	151.50	0.3637
19	159.92	0.3966
20	168.33	0.4299
21	176.75	0.4635
22	185.16	0.4974
23	193.58	0.5314
24	202.00	0.5656
25	212.46	0.6001
26	219.69	0.6342
27	227.73	0.6684

TABLE 8

DEFLECTION DUE TO EARTHQUAKE LOAD IN NORTH-SOUTH DIRECTION (WITH STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENTS (IN INCHES)
1	8.42	0.0016
2	16.83	0.0061
3	25.25	0.0133
4	33.67	0.0230
5	42.08	0.0351
6	50.50	0.0495
7	58.90	0.0660
8	67.33	0.0843
9	75.75	0.1041
10	84.16	0.1254
11	92.58	0.1479
12	101.00	0.1711
13	109.42	0.1952
14	117.83	0.2197
15	126.25	0.2445
16	134.66	0.2695
17	143.08	0.2943
18	151.50	0.3189
19	159.92	0.3339
20	168.33	0.3667
21	176.75	0.3895
22	186.16	0.4116
23	193.58	0.4326
24	202.00	0.4526
25	210.46	0.4715
26	219.69	0.4892
27	227.73	0.5127

TABLE 9

DEFLCTION DUE TO EARTHQUADE LOAD IN NORTH-SOUTH DIRECTION (WITHOUT STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0021
2	16.83	0.0081
3	25.25	0.0187
4	33.67	0.0312
5	42.08	0.0479
6	50.50	0.0682
7	58.92	0.0916
8	67.33	0.1180
9	75.75	0.1472
10	84.16	0.1789
11	92.58	0.2130
12	101.00	0.2492
13	109.42	0.2874
14	117.83	0.3271
15	126.25	0.3686
16	134.66	0.4112
17	143.08	0.4550
18	151.50	0.4998
19	159.92	0.5454
20	168.33	0.5917
21	176.75	0.6385
22	185.16	0.6856
23	193.58	0.7332
24	202.00	0.7809
25	210.46	0.8288
26	218.83	0.8768
27	231.06	0.9248

TABLE 10

FUNDAMENTAL MODE SHAPE IN EAST-WEST DIRECTION (WITH STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0641
2	16.83	0.2484
3	25.25	0.5416
4	33.67	0.9324
5	42.08	1.4143
6	50.50	1.9822
7	58.90	2.6248
8	67.33	3.3306
9	75.75	4.0923
10	84.16	4.8991
11	92.58	5.7445
12	101.00	6.6196
13	109.42	7.5159
14	117.83	8.4247
15	126.25	9.3411
16	134.66	10.2566
17	143.08	11.1676
18	151.50	12.0676
19	159.92	12.9518
20	168.33	13.8151
21	176.75	14.6564
22	185.16	15.4713
23	193.58	16.2602
24	202.00	17.0186
25	210.46	17.7406
26	219.69	18.4568
27	227.73	19.0369

TABLE 11

FUNDAMENTAL MODE SHAPE IN EAST-WEST DIRECTION (WITHOUT STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0832
2	16.83	0.3241
3	25.25	0.7112
4	33.67	1.2327
5	42.08	1.8837
6	50.50	2.6611
7	58.90	3.5529
8	67.33	4.5470
9	75.75	5.6367
10	84.16	6.8101
11	92.58	8.0621
12	101.00	9.3836
13	109.42	10.7652
14	117.83	12.1969
15	126.25	13.6750
16	134.66	15.1890
17	143.08	16.7365
18	151.50	18.3101
19	159.92	19.9046
20	168.33	21.5136
21	176.75	23.1370
22	185.16	24.7673
23	193.58	26.4058
24	202.00	28.0479
25	210.46	29.6997
26	219.69	31.3342
27	227.73	32.9782

TABLE 12

FUNDAMENTAL MODE SHAPE IN NORTH-SOUTH DIRECTION (WITH STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0004
2	16.83	0.0016
3	25.25	0.0034
4	33.67	0.0059
5	42.08	0.0091
6	50.50	0.0127
7	58.90	0.0170
8	67.33	0.0216
9	75.75	0.0267
10	84.16	0.0321
11	92.58	0.0378
12	101.00	0.0437
13	109.42	0.0498
14	117.83	0.0561
15	126.25	0.0624
16	134.66	0.0687
17	143.08	0.0750
18	151.50	0.0813
19	159.92	0.0874
20	168.33	0.0935
21	176.75	0.0993
22	185.16	0.1050
23	193.58	0.1105
24	202.00	0.1157
25	210.46	0.1207
26	219.73	0.1254
27	227.73	0.1318

TABLE 13

FUNDAMENTAL MODE SHAPE IN NORTH-SOUTH DIRECTION (WITHOUT STIFF BEAM)

FLOOR	ELEVATION (IN FEET)	HORIZONTAL DISPLACEMENT (IN INCH)
1	8.42	0.0003
2	16.83	0.0013
3	25.25	0.0030
4	33.67	0.0051
5	42.08	0.0078
6	50.50	0.0111
7	58.90	0.0148
8	67.33	0.0190
9	75.75	0.0235
10	84.16	0.0284
11	92.58	0.0368
12	101.00	0,0392
13	109.42	0.0450
14	117.83	0.0509
15	126.25	0.0570
16	134.66	0.0633
17	143.08	0.0697
18	151.50	0.0761
19	159.92	0.0826
20	168.33	0.0892
21	176.75	0.0958
22	185.16	0.1023
23	193.58	0.1089
24	202.00	0.1155
25	210.46	0.1221
26	219.73	0.1286
27	227.73	0.1351